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April 30, 2004

JN 03490

Rainier Commons, LLC
3100 Airport Way South
Seattle, Washington 98134

Attention: Shimone Mizrahi

Subject: **Transmittal Letter – Geotechnical Engineering Study**
Proposed Rainier Commons
3100 Airport Way South
Seattle, Washington

Dear Mr. Mizrahi:

We are pleased to present this geotechnical engineering report for the proposed Rainier Commons development to be constructed in Seattle, Washington. The scope of our services consisted of exploring site surface and subsurface conditions, and then developing this report to provide recommendations for general earthwork and design criteria for foundations, retaining walls, and temporary shoring. This work was authorized by your acceptance of our proposal, P-6243, dated December 8, 2003.

The attached report contains a discussion of the study and our recommendations. Please contact us if there are any questions regarding this report, or for further assistance during the design and construction phases of this project.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

D. Robert Ward, P.E.
Principal

cc: **LDG Architects** – Ed Linardic
via facsimile: (206) 283-1293

GDB/DRW: esn

GEOTECHNICAL ENGINEERING STUDY
Proposed Rainier Commons
3100 Airport Way South
Seattle, Washington

This report presents the findings and recommendations of our geotechnical engineering study for the site of the proposed Rainier Commons development to be located in Seattle.

We were provided with a faxed preliminary site plan of the proposed development. LDG Architects prepared this plan, which is undated. Based on this plan and conversations with Ed Linardic of LDG Architects, we understand that a new building is proposed at the southeastern side of the developed site. The proposed building will have an approximate finish floor elevation of 13 feet. Cuts of up to 35 feet are anticipated to reach the planned excavation bottom for the southern building. The deeper cuts for this building will be located along the eastern side of the excavation adjacent to Interstate 5.

If the scope of the project changes from what we have described above, we should be provided with revised plans in order to determine if modifications to the recommendations and conclusions of this report are warranted.

SITE CONDITIONS

SURFACE

The Vicinity Map, Plate 1, illustrates the general location of the site. The irregular-shaped property is located along the eastern side of Airport Way South in the SODO district of Seattle and is currently occupied by several buildings. The property used to be the Rainier Brewery facility. This facility is now comprised of office and manufacturing buildings.

The proposed building location covered in this study is located at the southeast corner of the property (eastern terminus of South Horton Street). The southeast portion of the site, located directly upslope from the existing cooling towers, is currently undeveloped. This area generally slopes from northeast to southwest at a moderate-to-steep rate. The slope is discontinuous, as some excavated trails and slopes were made on the site.

Bordering the entire facility along its perimeter are South Horton Street, Airport Way South, and South Stevens Street to the south, west, and north, respectively. Adjacent to the eastern side is the southbound offramp of Interstate 5 that exits to South Spokane Street and South Columbian Way. This overhead ramp is approximately 30 to 70 feet above existing grade where it is adjacent to the subject site. The ramp is a deck suspended over concrete columns. The ground beneath the ramp slopes moderately upward to the east. A more detailed description of Interstate 5 structures adjacent to the proposed building area is provided below (or on the next page).

Review of As-Built Plans and Related Information for Interstate 5

As part of our study, we reviewed the as-built plans and field reports for the existing Interstate 5/South Spokane Street Interchange. We viewed this information at the regional Washington State Department of Transportation (WSDOT) office in Shoreline and at the Engineering Records and Plans Vault located at the WSDOT head office in Olympia. Based on a review of the plans and field reports and conversations with WSDOT personnel, the interchange ramp is supported on above-ground columns that also extend into the ground. These columns/piers are located about 30 to 50 feet east of the eastern side of the proposed building on the subject site. The approximate depth of the concrete piers located adjacent to the proposed building range from 63 to 83 feet below existing grade. This would correspond to an approximate elevation of –15 to –35 feet. Therefore, it appears that the bottom of the piers for the ramp located adjacent to the proposed building is below the proposed bottom-of-excavation. The diameter of the piers (below grade) ranges from 3.5 to 6 feet. The plans also show that a sewer line approximately 108 inches in diameter runs parallel to the northern edge of the proposed building footprint. This sewer line also appears to be located below the proposed bottom-of-excavation.

A detailed description of the design loads for the columns did not appear to be listed in the field reports reviewed at the State DOT. However, four test piles were installed on the northeast corner of the interchange site prior to construction. These piles were drilled to depths of 30 to 50 feet below existing grade and each loaded to a maximum load of 500 tons.

SUBSURFACE

The subsurface conditions were explored by drilling three borings at the approximate locations shown on the Site Exploration Plan, Plate 2. Our exploration program was based on the proposed construction, anticipated subsurface conditions and those encountered during exploration, and the scope of work outlined in our proposal.

The borings were drilled on December 29, 2003, using a trailer-mounted, hollow-stem auger drill and portable Acker drill. The Acker drill system utilizes a small, gasoline-powered engine to advance a hollow-stem auger to the sampling depth. Samples were taken at 2.5- and 5-foot intervals with a standard penetration sampler. This split-spoon sampler, which has a 2-inch outside diameter, is driven into the soil with a 140-pound hammer falling 30 inches. The number of blows required to advance the sampler a given distance is an indication of the soil density or consistency. A geotechnical engineer from our staff observed the drilling process, logged the test borings, and obtained representative samples of the soil encountered. The Test Boring Logs are attached as Plates 3 through 7.

Soil Conditions

The three borings generally encountered similar subsurface conditions. The borings each encountered from 2 to 3 feet of loose gravelly sand fill overlying native, medium-stiff low-plasticity silts and medium-dense non-plastic silt. These silt soils became medium-dense to dense at a depth of 20 to 25 feet below existing grade, to the maximum explored depth of the borings. Boring 1 was explored to a depth of 36.5 feet below existing grade. Borings 2 and 3 were each explored to a depth of 45 feet below existing grade.

We obtained soil information from the City of Seattle. We found that one boring was previously drilled in the west central portion of the site in 1996. This boring generally encountered approximately 7.5 feet of loose fill overlying loose sand and medium-stiff silt. The silt then became stiff to very stiff at a depth of 20 feet below existing grade. The boring was explored to a depth of 36.5 feet below existing grade.

Numerous borings were conducted by the State of Washington prior to the construction of the Interstate 5/South Spokane Street Interchange. We obtained this information from the Washington State Department of Transportation. Three borings were drilled underneath or adjacent to the now-existing overhead ramp located east of the subject site. These borings generally encountered from 20 to 30 feet of stiff to very stiff silt overlying hard silt.

No obstructions were revealed by our explorations. However, debris, buried utilities, and old foundation and slab elements are commonly encountered on sites that have had previous development.

Groundwater Conditions

Groundwater seepage was observed at a depth of 40 feet in Boring 3. The borings were left open for only a short time period. Therefore, the seepage levels on the logs represent the location of transient water seepage and may not indicate the static groundwater level. Groundwater levels encountered during drilling can be deceptive, because seepage into the boring can be blocked or slowed by the auger itself.

It should be noted that groundwater levels vary seasonally with rainfall and other factors. We anticipate that groundwater could be found at somewhat higher levels in wet winter and spring periods.

The final logs represent our interpretations of the field logs. The stratification lines on the logs represent the approximate boundaries between soil types at the exploration locations. The actual transition between soil types may be gradual, and subsurface conditions can vary between exploration locations. The logs provide specific subsurface information only at the locations tested. If a transition in soil type occurred between samples in the borings, the depth of the transition was interpreted. The relative densities and moisture descriptions indicated on the boring logs are interpretive descriptions based on the conditions observed during drilling.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

THIS SECTION CONTAINS A SUMMARY OF OUR STUDY AND FINDINGS FOR THE PURPOSES OF A GENERAL OVERVIEW ONLY. MORE SPECIFIC RECOMMENDATIONS AND CONCLUSIONS ARE CONTAINED IN THE REMAINDER OF THIS REPORT. ANY PARTY RELYING ON THIS REPORT SHOULD READ THE ENTIRE DOCUMENT.

The borings conducted for this study at the southeast corner of the site generally encountered medium-dense silts within 10 feet below existing grade, which then became medium-dense to dense 20 to 25 feet below existing grade. It is our opinion that the proposed building can be supported on conventional foundations bearing on the medium-dense to dense, native soils. This soil will likely be exposed at the proposed excavation level. However, the on-site soils are silty, and thus are moisture sensitive. It may be necessary to protect bearing surfaces with a thin layer of crushed rock to protect the subgrades from disturbance during periods of wet weather. The on-site soils also have high moisture contents. Thus, these soils should not be reused as structural fill or free-draining backfill. However, we recommend that the wall be designed for one inch or less of horizontal movement so that the piers do not move laterally.

One of the most significant geotechnical engineering aspects for this project is the large excavation cuts, up to 35 feet in height, mostly near the eastern property line. The site soils can only be excavated to a maximum temporary slope of 1:1 (H:V). Where a temporary open excavation would extend beyond the property line or undermine an adjacent building, shoring should be used. The shoring can either be permanent or temporary, as discussed in a subsequent section of this report. Where the shoring is generally greater than 15 feet in height, tie-back anchors or internal braces will be needed. Tie-back anchors are preferred due to less potential for lateral movement of the wall, but an easement from the WSDOT must be obtained to install anchors into the area below the offramp.

Based on a review of the as-built drawings for the southbound Interstate 5 offramp to South Columbian Way and South Spokane Street, the bottom-of-pier elevation for each of the adjacent piles appears to be below the proposed bottom-of-excavation. Thus, no surcharge load will need to be included in the design of the eastern shoring wall.

Based on conversations with WSDOT personnel, because the wall would be situated adjacent to this offramp, WSDOT must review the shoring plans for the proposed building prior to construction. In addition, a representative from WSDOT will need to observe the construction of the shoring wall adjacent to the Interstate 5 right-of-way.

If the structure includes an elevator, it may be necessary to provide special drainage or waterproofing measures for the elevator pit. If no seepage into the elevator pit is acceptable, it will be necessary to provide a footing drain and free-draining wall backfill, and the walls should be waterproofed. If the footing drain will be too low to connect to the storm drainage system, then it will likely be necessary to install a pumped sump to discharge the collected water. Alternatively, the elevator pit could be designed to be entirely waterproof; this would include designing the pit structure to resist hydrostatic uplift pressures.

The erosion control measures needed during the site development will depend heavily on the weather conditions that are encountered. While site clearing will expose a large area of bare soil, the erosion potential on the site is relatively low. We anticipate that a silt fence will be needed around the downslope sides of any cleared areas. Rocked construction access roads should be extended into the site to reduce the amount of soil or mud carried off the property by trucks and equipment. Wherever possible, these roads should follow the alignment of planned pavements. Cut slopes and soil stockpiles should be covered with plastic during wet weather.

Geotech Consultants, Inc. should be allowed to review the final development plans to verify that the recommendations presented in this report are adequately addressed in the design. Such a plan review would be additional work beyond the current scope of work for this study, and it may include revisions to our recommendations to accommodate site, development, and geotechnical constraints that become more evident during the review process.

We recommend including this report, in its entirety, in the project contract documents. This report should also be provided to any future property owners so they will be aware of our findings and recommendations.

SEISMIC CONSIDERATIONS

The site is located within Seismic Zone 3, as illustrated on Figure No. 16-2 of the 1997 Uniform Building Code (UBC). In accordance with Table 16-J of the 1997 UBC, the site soil profile within 100 feet of the ground surface is best represented by Soil Profile Type S_D (Stiff Soil). The site soils are not susceptible to seismic liquefaction because of their dense nature.

CONVENTIONAL FOUNDATIONS

The proposed structure can be supported on conventional continuous and spread footings bearing on undisturbed, medium-dense to dense native soil. We recommend that continuous and individual spread footings have minimum widths of 16 and 24 inches, respectively. Footings should also be bottomed at least 18 inches below the lowest adjacent finish ground surface. The local building codes should be reviewed to determine if different footing widths or embedment depths are required. Footing subgrades must be cleaned of loose or disturbed soil prior to pouring concrete. Depending upon site and equipment constraints, this may require removing the disturbed soil by hand.

An allowable bearing pressure of 5,000 pounds per square foot (psf) is appropriate for footings supported on competent native soil. A one-third increase in this design bearing pressure may be used when considering short-term wind or seismic loads. For the above design criteria, it is anticipated that the total post-construction settlement of footings founded on competent native soil will be less than 1 inch, with differential settlement in the range of 3/4 inch over a 75-foot length of foundation.

Lateral loads due to wind or seismic forces may be resisted by friction between the foundation and the bearing soil, or by passive earth pressure acting on the vertical, embedded portions of the foundation. For the latter condition, the foundation must be either poured directly against relatively level, undisturbed soil or be surrounded by level structural fill. We recommend using the following ultimate values for the foundation's resistance to lateral loading:

PARAMETER	ULTIMATE VALUE
Coefficient of Friction	0.40
Passive Earth Pressure	300 pcf

Where: (i) pcf is pounds per cubic foot, and (ii) passive earth pressure is computed using the equivalent fluid density.

If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate. We recommend maintaining a safety factor of at least 1.5 for the foundation's resistance to lateral loading, when using the above ultimate values.

PERMANENT FOUNDATION AND RETAINING WALLS

Retaining walls backfilled on only one side should be designed to resist the lateral earth pressures imposed by the soil they retain. The following are recommended parameters for retaining walls:

PARAMETER	VALUE
Active Earth Pressure * - level backslope	40 pcf
Active Earth Pressure * - eastern foundation wall	55 pcf
Passive Earth Pressure	300 pcf
Coefficient of Friction	0.40
Soil Unit Weight	130 pcf

Where: (i) pcf is pounds per cubic foot, and (ii) active and passive earth pressures are computed using the equivalent fluid pressures.

*** For a restrained wall that cannot deflect at least 0.002 times its height, a uniform lateral pressure equal to 10 psf times the height of the wall should be added to the above active equivalent fluid pressure.**

The values given above are to be used to design permanent foundation and retaining walls only. The passive pressure given is appropriate for the depth of level structural fill placed in front of a retaining or foundation wall only. The values for friction and passive resistance are ultimate values and do not include a safety factor. We recommend a safety factor of at least 1.5 for overturning and sliding, when using the above values to design the walls. Restrained wall soil parameters should be utilized for a distance of 1.5 times the wall height from corners or bends in the walls. This is intended to reduce the amount of cracking that can occur where a wall is restrained by a corner.

The design values given above do not include the effects of any hydrostatic pressures behind the walls and assume that no surcharges, such as those caused by vehicles or adjacent foundations will be exerted on the walls. If these conditions exist, those pressures should be added to the above lateral soil pressures. The surcharge due to traffic loads behind a wall can typically be accounted for by adding a uniform pressure equal to 2 feet multiplied by the above active fluid density.

Heavy construction equipment should not be operated behind retaining and foundation walls within a distance equal to the height of a wall, unless the walls are designed for the additional lateral pressures resulting from the equipment. The wall design criteria assume that the backfill will be well-compacted in lifts no thicker than 12 inches. The compaction of backfill near the walls should be accomplished with hand-operated equipment to prevent the walls from being overloaded by the higher soil forces that occur during compaction.

Retaining Wall Backfill

Backfill placed behind retaining or foundation walls should be coarse, free-draining structural fill containing no organics. This backfill should contain no more than 5 percent silt or clay particles and have no gravel greater than 4 inches in diameter. The percentage of particles passing the No. 4 sieve should be between 25 and 70 percent. The on-site, native soils should not be reused as wall backfill due to their silty nature. The later section entitled ***Drainage Considerations*** should also be reviewed for recommendations related to subsurface drainage behind foundation and retaining walls.

The purpose of these backfill requirements is to ensure that the design criteria for a retaining wall are not exceeded because of a build-up of hydrostatic pressure behind the wall. The top 12 to 18 inches of the backfill should consist of a compacted, relatively impermeable soil or topsoil, or the surface should be paved. The ground surface must also slope away from backfilled walls to reduce the potential for surface water to percolate into the backfill. The section entitled ***General Earthwork and Structural Fill*** contains recommendations regarding the placement and compaction of structural fill behind retaining and foundation walls.

The above recommendations are not intended to waterproof below-grade walls, or to prevent the formation of mold, mildew or fungi in interior spaces. Over time, the performance of subsurface drainage systems can degrade, subsurface groundwater flow patterns can change, and utilities can break or develop leaks. Therefore, waterproofing should be provided where future seepage through the walls is not acceptable. This typically includes limiting cold-joints and wall penetrations, and using bentonite panels or membranes on the outside of the walls. There are a variety of different waterproofing materials and systems, which should be installed by an experienced contractor familiar with the anticipated construction and subsurface conditions. Applying a thin coat of asphalt emulsion to the outside face of a wall is not considered waterproofing, and will only help to reduce moisture generated from water vapor or capillary action from seeping through the concrete. As with any project, adequate ventilation of basement and crawl space areas is important to prevent a build up of water vapor that is commonly transmitted through concrete walls from the surrounding soil, even when seepage is not present. This is appropriate even when waterproofing is applied to the outside of foundation and retaining walls. We recommend that you contact a specialty consultant if detailed recommendations or specifications related to waterproofing design, or minimizing the potential for infestations of mold and mildew are desired.

The ***General***, ***Slabs-On-Grade***, and ***Drainage Considerations*** sections should be reviewed for additional recommendations related to the control of groundwater and excess water vapor for the anticipated construction.

SHORING

This section presents design considerations for cantilevered or tied-back soldier-pile walls. Since the most suitable choice is primarily dependent on a number of factors under the contractor's control, we suggest that the contractor work closely with the structural engineer during the shoring design. As noted earlier, the shoring design should include a maximum of one inch of deflection. In addition, the shoring could be considered temporary or permanent, however, we assume that

WSDOT would not approve of permanent tie-back anchors in state property, thus only temporary design parameters are given for anchors.

The shoring design should be submitted to Geotech Consultants, Inc. for review prior to beginning site excavation. We are available and would be pleased to assist in this design effort. In addition, as noted earlier, WSDOT requires that it review the shoring design prior to construction.

Cantilevered and Tied-Back Soldier Piles

Cantilevered and tied-back soldier-pile systems have proven to be an efficient and economical method for providing excavation shoring. Tied-back walls are typically more economical than cantilevered walls where the depth of excavation is greater than 15 feet.

Soldier Pile Installation

Soldier-pile walls would be constructed after making planned cut slopes, and prior to commencing the mass excavation, by setting steel H-beams in a drilled hole and grouting the space between the beam and the soil with concrete for the entire height of the drilled hole. We anticipate that the holes could be drilled without casing, but the contractor should be prepared to case the holes or use the slurry method if caving soil is encountered. Excessive ground loss in the drilled holes must be avoided to reduce the potential for settlement on adjacent properties. If water is present in a hole at the time the soldier pile is poured, concrete must be tremied to the bottom of the hole.

As excavation proceeds downward, the space between the piles should be lagged with treated timber, and any voids behind the timbers should be filled with pea gravel, or a slurry comprised of sand and fly ash. Temporary vertical cuts will be necessary between the soldier piles for the lagging placement. The prompt and careful installation of lagging is important, particularly in loose or caving soil, to maintain the integrity of the excavation and provide safer working conditions. Additionally, care must be taken by the excavator to remove no more soil between the soldier piles than is necessary to install the lagging. Caving or overexcavation during lagging placement could result in loss of ground on neighboring properties. Timber lagging should be designed for an applied lateral pressure of 30 percent of the design wall pressure, if the pile spacing is less than three pile diameters. For larger pile spacings, the lagging should be designed for 50 percent of the design load.

If permanent building walls are to be constructed against the shoring walls, drainage should be provided by attaching a geotextile drainage composite with a solid plastic backing, similar to Miradrain 6000, to the face of the lagging, prior to pouring the foundation wall. These drainage composites should be hydraulically connected to the foundation drainage system through weep holes placed in the foundation walls.

Soldier-Pile Wall Design

Temporary soldier-pile shoring that is cantilevered or restrained by one row of tiebacks, and that has a level backslope, should be designed for an active soil pressure equal to that pressure exerted by an equivalent fluid with a unit weight of 35 pounds per cubic foot (pcf). For sloped conditions on the eastern side of the site, the active pressure should be 50 pcf. If the wall is to be permanent, 5 pcf should be added to the 35 or 50 pcf pressure. To design temporary tied-back shoring with more than one row of tiebacks and a level backslope, we recommend assuming that the lateral active soil pressure on the wall, expressed in pounds per square foot (psf), is equal to 25**H** or 35**H**, depending on flat or sloped backslope conditions where **H** is the total height of the excavation in feet. Traffic surcharges can be accounted for by increasing the effective height of the shoring wall by 2 feet. Existing adjacent buildings will exert surcharges on the proposed shoring wall, if they extend within a 1:1 (H:V) slope below the building footing, unless the buildings are underpinned.

If tieback easements cannot be obtained, it would be necessary to utilize internal braces (rakers) to restrain the soldier piles. Soldier piles restrained by rakers typically undergo more deflection than do tied-back piles, due to the excavation that is necessary in front of the soldier piles to install the rakers and thrust blocks. This type of soldier-pile restraint requires that the shoring designer closely evaluate the temporary conditions that exist before raker installation. We should be contacted if rakers are necessary, in order to provide the appropriate design considerations.

It is important that the shoring design provides sufficient working room to drill and install the soldier piles, without needing to make unsafe, excessively-steep temporary cuts. Cut slopes should be planned to intersect the backside of the drilled holes, not the back of the lagging.

Lateral movement of the soldier piles below the excavation level will be resisted by an ultimate passive soil pressure equal to that pressure exerted by a fluid with a density of 450 pcf. No safety factor is included in the given value. This soil pressure is valid only for a level excavation in front of the soldier pile; it acts on two times the grouted pile diameter. Cut slopes made in front of shoring walls significantly decrease the passive resistance. This includes temporary cuts necessary to install internal braces or rakers. The minimum embedment below the floor of the excavation for cantilever soldier piles should be equal to the height of the "stick-up."

Tieback Anchors

General considerations for the design of tied-back or braced soldier pile walls are presented on Plate 9. We recommend installing tieback anchors at inclinations between 20 and 30 degrees below horizontal. The tieback will derive its capacity from the soil-grout strength developed in the soil behind the no-load zone. The no-load zone is the area behind which the entire length of each tieback anchor should be located.

To prevent excessive loss of ground in a drilled hole, the no-load section of the drilled tieback hole should be backfilled with a sand and fly ash slurry, after protecting the anchor with a bond breaker, such as plastic casing, to prevent loads from being transferred to the soil in the no-load zone. The no-load section could be filled with grout after anchor testing is completed.

During the design process, the possible presence of foundations or utilities close to the shoring wall must be evaluated to determine if they will affect the configuration and length of the tiebacks. The piers and sewer in the adjacent WSDOT property will have to be avoided, and the designer must review WSDOT as-built drawings.

Based on the results of our analyses and our experience at other construction sites, we suggest using an adhesion value of 1,200 psf in the dense sandy silt to design temporary anchors, if the mid-point of the grouted portion of the anchor is more than 15 feet below the overlying ground surface.

This value applies to non-pressure-grouted anchors. Pressure-grouted or post-grouted anchors can often develop adhesion values that are two to three times higher than that for non-pressure-grouted anchors. These higher adhesion values must be verified by load testing.

Soil conditions, soil-grout adhesion strengths, and installation techniques typically vary over any site. This sometimes results in adhesion values that are lower than anticipated. Therefore, we recommend substantiating the anchor design values by load-testing all tieback anchors. At least two anchors in each soil type encountered should be performance-tested to 200 percent of the design anchor load to evaluate possible anchor creep. Wherever possible, the no-load section of these tiebacks should not be grouted until the performance tests are completed. Unfavorable results from these performance tests could require increasing the lengths of the tiebacks. The remaining anchors should be proof-tested to at least 135 percent of their design value before being "locked off." After testing, each anchor should be locked off at a prestress load of 80 to 100 percent of its design load.

If caving or water-bearing soil is encountered, the installation of tieback anchors will be hampered by caving and soil flowing into the holes. It will be necessary to case the holes, if such conditions are encountered. Alternatively, the use of a hollow-stem auger with grout pumped through the stem as the auger is withdrawn would be satisfactory, provided that the injection pressure and grout volumes pumped are carefully monitored.

All drilled installations should be grouted and backfilled immediately after drilling. No drilled holes should be left open overnight.

SLABS-ON-GRADE

The building floors may be constructed as slabs-on-grade atop non-organic native soils, or on structural fill. The subgrade soil must be in a firm, non-yielding condition at the time of slab construction or underslab fill placement. Any soft areas encountered should be excavated and replaced with select, imported structural fill.

All slabs-on-grade should be underlain by a capillary break or drainage layer consisting of a minimum 4-inch thickness of coarse, free-draining structural fill with a gradation similar to that discussed in ***Permanent Foundation and Retaining Walls***. As noted by the American Concrete Institute (ACI) in the *Guides for Concrete Floor and Slab Structures*, proper moisture protection is desirable immediately below any on-grade slab that will be covered by tile, wood, carpet, impermeable floor coverings, or any moisture-sensitive equipment or products. ACI also notes that vapor *retarders*, such as 6-mil plastic sheeting, are typically used. A vapor retarder is defined as a material with a permeance of less than 0.3 US perms per square foot (psf) per hour, as determined by ASTM E 96. It is possible that concrete admixtures may meet this specification, although the manufacturers of the admixtures should be consulted. Where plastic sheeting is used under slabs, joints should overlap by at least 6 inches and be sealed with adhesive tape. The sheeting should extend to the foundation walls for maximum vapor protection. If no potential for vapor passage through the slab is desired, a vapor *barrier* should be used. A vapor barrier, as defined by ACI, is a product with a water transmission rate of 0.00 perms per square foot per hour when tested in accordance with ASTM E 96. Reinforced membranes having sealed overlaps can meet this requirement.

In the recent past, ACI (Section 4.1.5) recommended that a minimum of 4 inches of well-graded compactable granular material, such as a 5/8 inch minus crushed rock pavement base, should be placed over the vapor retarder or barrier for protection of the retarder or barrier and as a "blotter" to aid in the curing of the concrete slab. Sand was not recommended by ACI for this purpose. However, the use of material over the vapor retarder is controversial as noted in current ACI literature because of the potential that the protection/blotter material can become wet between the time of its placement and the installation of the slab. If the material is wet prior to slab placement, which is always possible in the Puget Sound area, it could cause vapor transmission to occur up through the slab in the future, essentially destroying the purpose of the vapor barrier/retarder. Therefore, if there is a potential that the protection/blotter material will become wet before the slab is installed, ACI now recommends that no protection/blotter material be used. However, ACI then recommends that, because there is a potential for slab cure due to the loss of the blotter material, joint spacing in the slab be reduced, a low shrinkage concrete mixture be used, and "other measures" (steel reinforcing, etc.) be used. ASTM E-1643-98 "Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs" generally agrees with the recent ACI literature.

We recommend that the contractor, the project materials engineer, and the owner discuss these issues and review recent ACI literature and ASTM E-1643 for installation guidelines and guidance on the use of the protection/blotter material. Our opinion is that with impervious surfaces that all means should be undertaken to reduce water vapor transmission.

The ***General***, ***Permanent Foundation and Retaining Walls***, and ***Drainage Considerations*** sections should be reviewed for additional recommendations related to the control of groundwater and excess water vapor for the anticipated construction.

EXCAVATIONS AND SLOPES

Excavation slopes should not exceed the limits specified in local, state, and national government safety regulations. Temporary cuts to a depth of about 4 feet may be attempted vertically in unsaturated soil, if there are no indications of slope instability. However, vertical cuts should not be made near property boundaries, or existing utilities and structures. Based upon Washington Administrative Code (WAC) 296, Part N, the soil at the subject site would generally be classified as

Type B. Therefore, temporary cut slopes greater than 4 feet in height cannot be excavated at an inclination steeper than 1:1 (Horizontal:Vertical), extending continuously between the top and the bottom of a cut.

The above-recommended temporary slope inclination is based on what has been successful at other sites with similar soil conditions. Temporary cuts are those that will remain unsupported for a relatively short duration to allow for the construction of foundations, retaining walls, or utilities. Temporary cut slopes should be protected with plastic sheeting during wet weather. The cut slopes should also be backfilled or retained as soon as possible to reduce the potential for instability. Please note that loose soil can cave suddenly and without warning. Excavation, foundation, and utility contractors should be made especially aware of this potential danger.

All permanent cuts into native soil should be inclined no steeper than 2:1 (H:V). Fill slopes should also not be constructed with an inclination greater than 2:1 (H:V).

To reduce the potential for shallow sloughing, fill must be compacted to the face of these slopes. This can be accomplished by overbuilding the compacted fill and then trimming it back to its final inclination. Adequate compaction of the slope face is important for long-term stability and is necessary to prevent excessive settlement of patios, slabs, foundations, or other improvements that may be placed near the edge of the slope.

Water should not be allowed to flow uncontrolled over the top of any temporary or permanent slope. All permanently exposed slopes should be seeded with an appropriate species of vegetation to reduce erosion and improve the stability of the surficial layer of soil.

EXCAVATION AND SHORING MONITORING

As with any shoring system, there is a potential risk of greater-than-anticipated movement of the shoring and the ground outside of the excavation. This can translate into noticeable damage of surrounding on-grade elements, such as foundations and slabs. Therefore, we recommend making an extensive photographic and visual survey of the project vicinity, prior to demolition activities, installing shoring or commencing excavation. This documents the condition of buildings, pavements, and utilities in the immediate vicinity of the site in order to avoid, and protect the owner from, unsubstantiated damage claims by surrounding property owners.

Additionally, the shoring walls should be monitored during construction to detect soil movements. To monitor their performance, we recommend establishing a series of survey reference points to measure any horizontal deflections of the shoring system. Control points should be established at a distance well away from the walls and slopes, and deflections from the reference points should be measured throughout construction by survey methods. At least every third soldier pile should be monitored by taking readings at the top of the pile. Additionally, the WSDOT ramp piers should be monitored for potential vertical and horizontal movement. We suggest taking the readings at least once a week, until it is established that no deflections are occurring. The initial readings for this monitoring should be taken before starting any demolition or excavation on the site.

DRAINAGE CONSIDERATIONS

We anticipate that the foundation walls will be constructed against the shoring walls. Where this occurs, a drainage composite should be placed against the lagging prior to pouring the foundation wall. Weep pipes located no more than 6-feet-on-center should be connected to the drainage composite and poured into the foundation walls or the perimeter footing. A footing drain installed along the inside of the perimeter footing will be used to collect and carry the water discharged by the weep pipes to the storm system. Footing drains placed inside the building or behind backfilled walls should consist of 4-inch, perforated PVC pipe surrounded by at least 6 inches of 1-inch-minus, washed rock wrapped in a non-woven, geotextile filter fabric (Mirafi 140N, Supac 4NP, or similar material). At its highest point, a perforated pipe invert should be at least as low as the bottom of the footing, and it should be sloped for drainage. All roof and surface water drains must be kept separate from the foundation drain system.

Foundation drains should also be placed at the base of all other non-shored foundations and earth-retaining walls. These drains should be constructed in the same fashion as described above. A typical drain detail is attached to this report as Plate 8. For the best long-term performance, perforated PVC pipe is recommended for all subsurface drains. The City of Seattle typically requires that Schedule 40 PVC pipe be used beneath structures.

If the structure includes an elevator, it may be necessary to provide special drainage or waterproofing measures for the elevator pit. If no seepage into the elevator pit is acceptable, it will be necessary to provide a footing drain and free-draining wall backfill, and the walls should be waterproofed. If the footing drain will be too low to connect to the storm drainage system, then it will likely be necessary to install a pumped sump to discharge the collected water. Alternatively, the elevator pit could be designed to be entirely waterproof; this would include designing the pit structure to resist hydrostatic uplift pressures.

Drainage inside the building's footprint should also be provided if the excavation encounters significant seepage. We can provide recommendations for interior drains, should they become necessary, during excavation and foundation construction.

As a minimum, a vapor retarder, as defined in the ***Slabs-On-Grade*** section, should be provided in any crawl space area to limit the transmission of water vapor from the underlying soils. Also, an outlet drain is recommended for all crawl spaces to prevent a build up of any water that may bypass the footing drains.

Groundwater was observed during our field work. If seepage is encountered in an excavation, it should be drained from the site by directing it through drainage ditches, perforated pipe, or French drains, or by pumping it from sumps interconnected by shallow connector trenches at the bottom of the excavation.

The excavation and site should be graded so that surface water is directed off the site and away from the tops of slopes. Water should not be allowed to stand in any area where foundations, slabs, or pavements are to be constructed. Final site grading in areas adjacent to buildings should slope away at least 2 percent, except where the area is paved. Surface drains should be provided where necessary to prevent ponding of water behind foundation or retaining walls.

GENERAL EARTHWORK AND STRUCTURAL FILL

All building and pavement areas should be stripped of surface vegetation, topsoil, organic soil, and other deleterious material. It is important that existing foundations be removed before site development. The stripped or removed materials should not be mixed with any materials to be used as structural fill, but they could be used in non-structural areas, such as landscape beds.

Structural fill is defined as any fill, including utility backfill, placed under, or close to, a building, behind permanent retaining or foundation walls, or in other areas where the underlying soil needs to support loads. All structural fill should be placed in horizontal lifts with a moisture content at, or near, the optimum moisture content. The optimum moisture content is that moisture content that results in the greatest compacted dry density. The moisture content of fill is very important and must be closely controlled during the filling and compaction process. As discussed in the ***General*** section, the on-site soils are not suitable for reuse as structural fill, due to their silty nature and high moisture contents.

The allowable thickness of the fill lift will depend on the material type selected, the compaction equipment used, and the number of passes made to compact the lift. The loose lift thickness should not exceed 12 inches.

We recommend testing the fill as it is placed. If the fill is not sufficiently compacted, it can be recompacted before another lift is placed. This eliminates the need to remove the fill to achieve the required compaction. The following table presents recommended relative compactions for structural fill:

LOCATION OF FILL PLACEMENT	MINIMUM RELATIVE COMPACTION
Beneath footings, slabs or walkways	95%
Filled slopes and behind retaining walls	90%

Where: Minimum Relative Compaction is the ratio, expressed in percentages, of the compacted dry density to the maximum dry density, as determined in accordance with ASTM Test Designation D 1557-91 (Modified Proctor).

The ***General*** section should be reviewed for a detailed discussion related to the reuse of on-site soils. Structural fill that will be placed in wet weather should consist of a coarse granular soil with a silt or clay content of no more than 5 percent. The percentage of particles passing the No. 200 sieve should be measured from that portion of soil passing the three-quarter-inch sieve.

<u>LIMITATIONS</u>

The analysis, conclusions, and recommendations contained in this report are based on site conditions as they existed at the time of our exploration and assume that the soil and groundwater conditions encountered in the borings are representative of subsurface conditions on the site. If the subsurface conditions encountered during construction are significantly different from those observed in our explorations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. Unanticipated soil conditions are commonly encountered on construction sites and cannot be fully anticipated by merely taking soil samples in borings. Subsurface conditions can also vary between exploration locations. Such unexpected conditions frequently require making additional expenditures to attain a properly constructed project. It is recommended that the owner consider providing a contingency fund to accommodate such potential extra costs and risks. This is a standard recommendation for all projects.

This report has been prepared for the exclusive use of Rainier Commons, LLC, and its representatives, for specific application to this project and site. Our recommendations and conclusions are based on observed site materials and selective laboratory testing. Our conclusions and recommendations are professional opinions derived in accordance with current standards of practice within the scope of our services and within budget and time constraints. No warranty is expressed or implied. The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design. Our services also do not include assessing or minimizing the potential for biological hazards, such as mold, bacteria, mildew and fungi in either the existing or proposed site development.

ADDITIONAL SERVICES

In addition to reviewing the final plans, Geotech Consultants, Inc. should be retained to provide geotechnical consultation, testing, and observation services during construction. This is to confirm that subsurface conditions are consistent with those indicated by our exploration, to evaluate whether earthwork and foundation construction activities comply with the general intent of the recommendations presented in this report, and to provide suggestions for design changes in the event subsurface conditions differ from those anticipated prior to the start of construction. However, our work would not include the supervision or direction of the actual work of the contractor and its employees or agents. Also, job and site safety, and dimensional measurements, will be the responsibility of the contractor.

During the construction phase, we will provide geotechnical observation and testing services only when requested by you or your representatives. We can only document site work that we actually observe. It is still the responsibility of your contractor or on-site construction team to verify that our recommendations are being followed, whether we are present at the site or not.

The scope of our work did not include an environmental assessment, but we can provide this service, if requested.

The following plates are attached to complete this report:

Plate 1	Vicinity Map
Plate 2	Site Exploration Plan
Plates 3 - 7	Boring Logs
Plate 8	Typical Footing Drain Detail
Plate 9	Tied-Back Shoring Detail

We appreciate the opportunity to be of service on this project. If you have any questions, or if we may be of further service, please do not hesitate to contact us.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

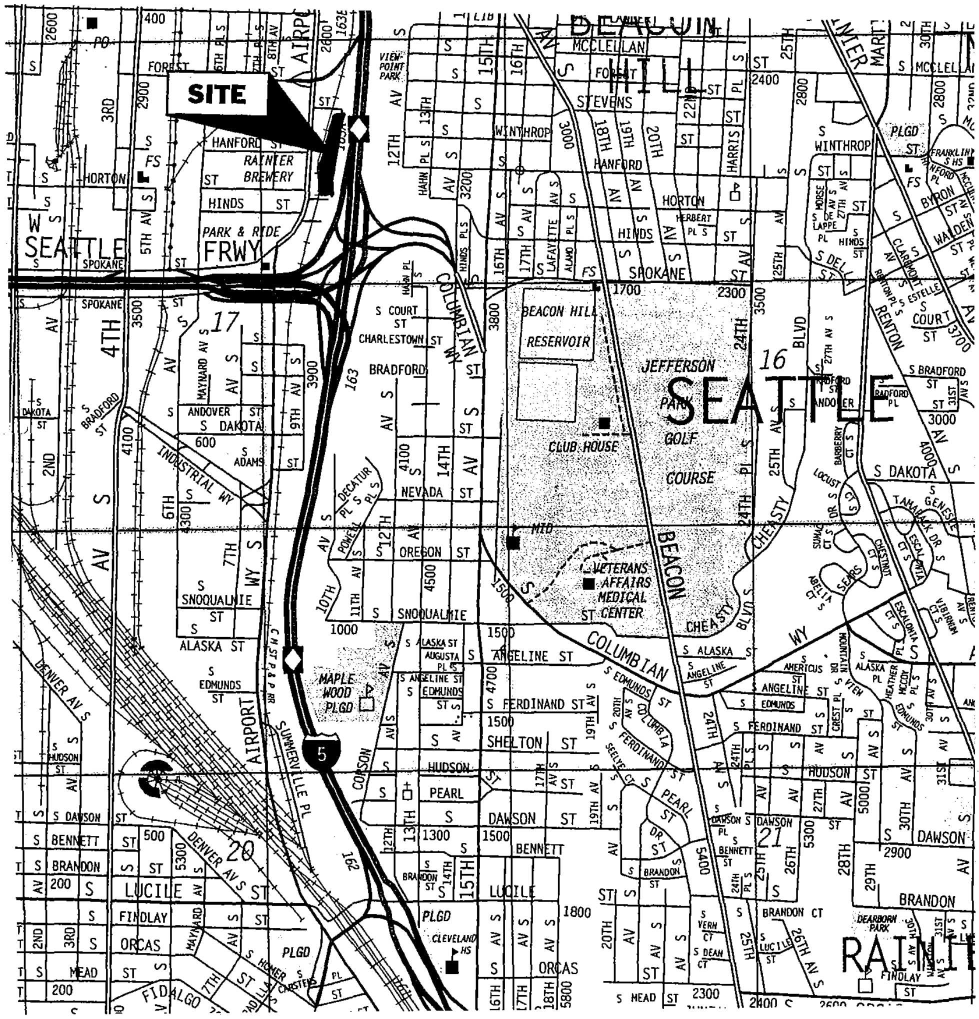
Gerry D. Bautista, Jr.
Geotechnical Engineer

D. ROBERT WARD
STATE OF WASHINGTON
REGISTERED
PROFESSIONAL ENGINEER

EXPIRES 10/21/05

D. Robert Ward, P.E.
Principal

GDB/DRW: esn



NORTH

(Source: Thomas Brothers Street Guide and Directory)

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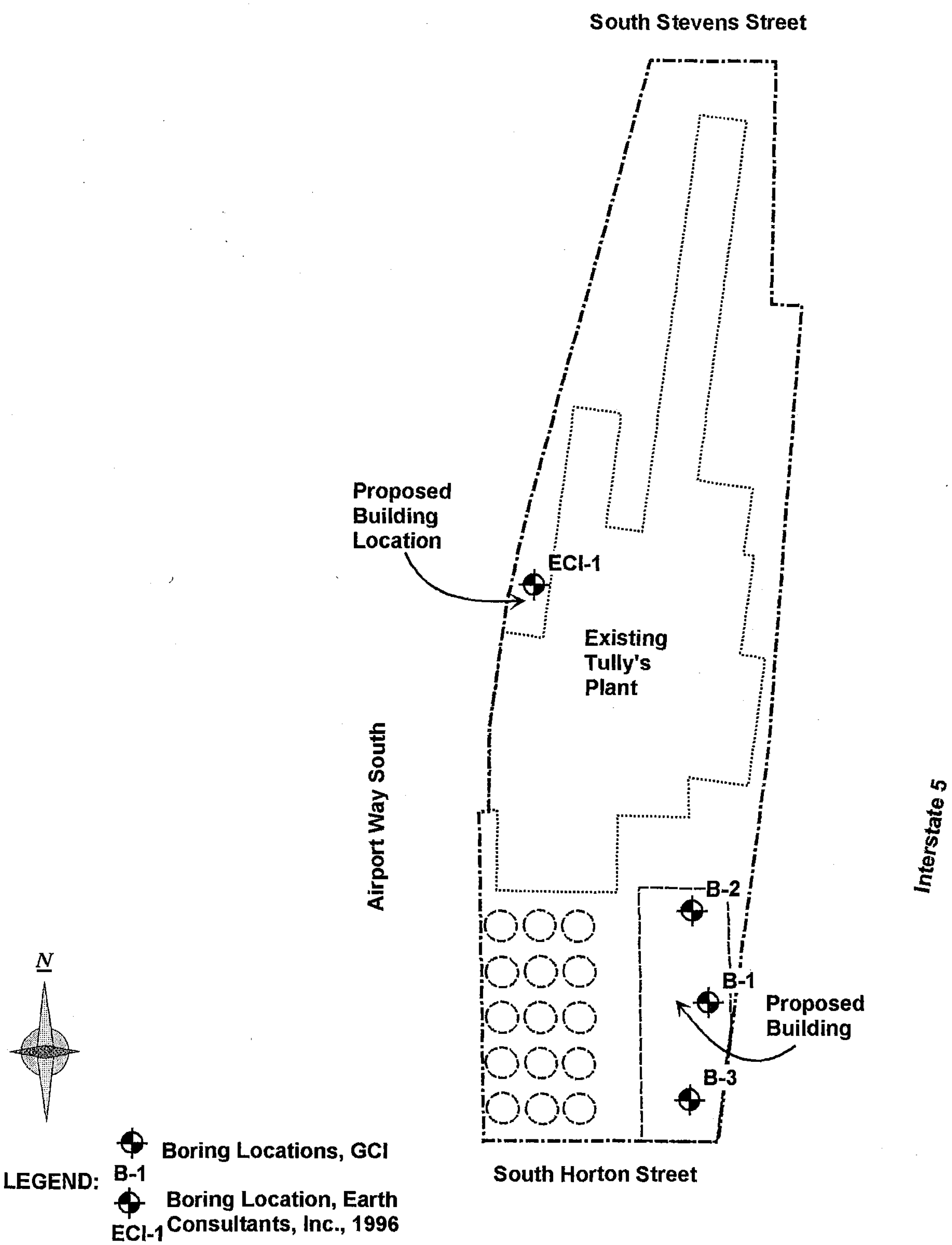
VICINITY MAP

3100 Airport Way South
Seattle, Washington

Job No:	Date:		Plate:
03490	Apr. 2004	Not To Scale	1



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SITE EXPLORATION PLAN

3100 Airport Way South
Seattle, Washington

Job No:	Date:		Plate:
03490	Apr. 2004	Not To Scale	2

BORING 1

Depth (ft)	Water Table	Blows per Foot	Sample	USCS	Description
0				FILL	Gray, gravelly, silty SAND, fine- to medium-grained, moist, loose (***FILL***)
				ML	Gray SILT, low plasticity, fractured, moist, soft to medium-stiff
5		7	1		
10		23	2		- becomes non-plastic, less fractured, medium-dense
15		27	3		- becomes slightly sandy
20		34	4		
25		22	5		- trace of clay
30		23	6		- becomes slightly sandy
35		41	7		- becomes dense
40					

*** Test boring was terminated at 36.5 feet during drilling on December 29, 2003.**
*** No groundwater seepage was encountered during drilling.**

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BORING LOG

3100 Airport Way South
Seattle, Washington

Job No:	Date:	Logged by:	Plate:
03490	Apr. 2004	GDB	3

BORING 2

Depth (ft)	Water Table	Blows per Foot	Sample	USCS	Description
				FILL	Gray, silty, gravelly SAND, fine- to medium-grained, moist, loose (***FILL***)
				ML	Gray, sandy SILT, non-plastic, moist, medium-dense
5		13	1		
10		17	2		
15		25	3		- becomes sandy
20		32	4		- becomes dense
25		28	5		- becomes less sandy, medium-dense
30		24	6		
35		44	7		- becomes dense
40		28	8		

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BORING LOG
3100 Airport Way South
Seattle, Washington

Job No:	Date:	Logged by:	Plate:
03490	Apr. 2004	GDB	4

BORING 2 - continued

Depth	Water Table	Blows per Foot	Sample	USCS	Description
				ML	
45		32	9		

* Test boring was terminated at 45.0 feet during drilling on December 29, 2003.
* No groundwater seepage was encountered during drilling.

50

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BORING LOG
3100 Airport Way South
Seattle, Washington

Job No:	Date:	Logged by:	Plate:
03490	Apr. 2004	GDB	5

BORING 3

Depth	Water Table	Blows per Foot	Sample	USCS	Description
				FILL	Gray, silty, gravelly SAND, fine- to medium-grained, moist, loose (***FILL***)
5		13	1	ML	Gray SILT, fractured, low plasticity, moist, stiff
10		23	2		- becomes less fractured, medium-dense (wood debris in sampler)
15		27	3		
20		38	4		- becomes slightly sandy, very moist, dense
25		30	5		
30		29	6		
35		25	7		
40	▼	40	8		- becomes wet, dense

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BORING LOG

3100 Airport Way South
Seattle, Washington

Job No:	***Date:***	***Logged by:***	***Plate:***
03490	Apr. 2004	GDB	6

BORING 3 - continued

Depth	Water Table	Blows per Foot	Sample	USCS	Description
45		40	9	ML	
50					

* Test boring was terminated at 45.0 feet during drilling on December 29, 2003.
* Groundwater seepage was encountered at 40 feet during drilling.

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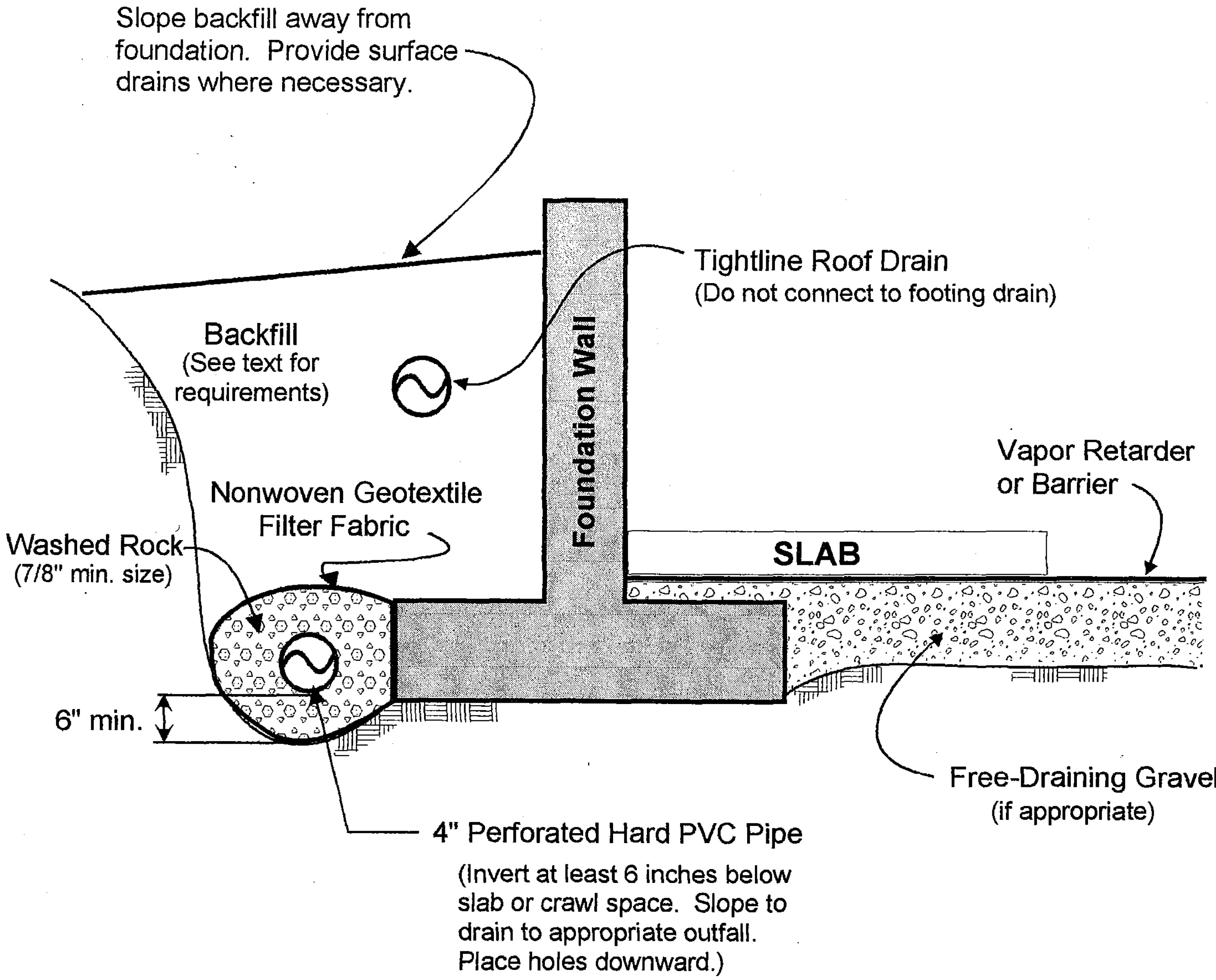
BORING LOG

3100 Airport Way South
Seattle, Washington

Job No:	Date:	Logged by:	Plate:
03490	Apr. 2004	GDB	7



NOTES:

(1) In crawl spaces, provide an outlet drain to prevent buildup of water that bypasses the perimeter footing drains.

(2) Refer to report text for additional drainage and waterproofing considerations.

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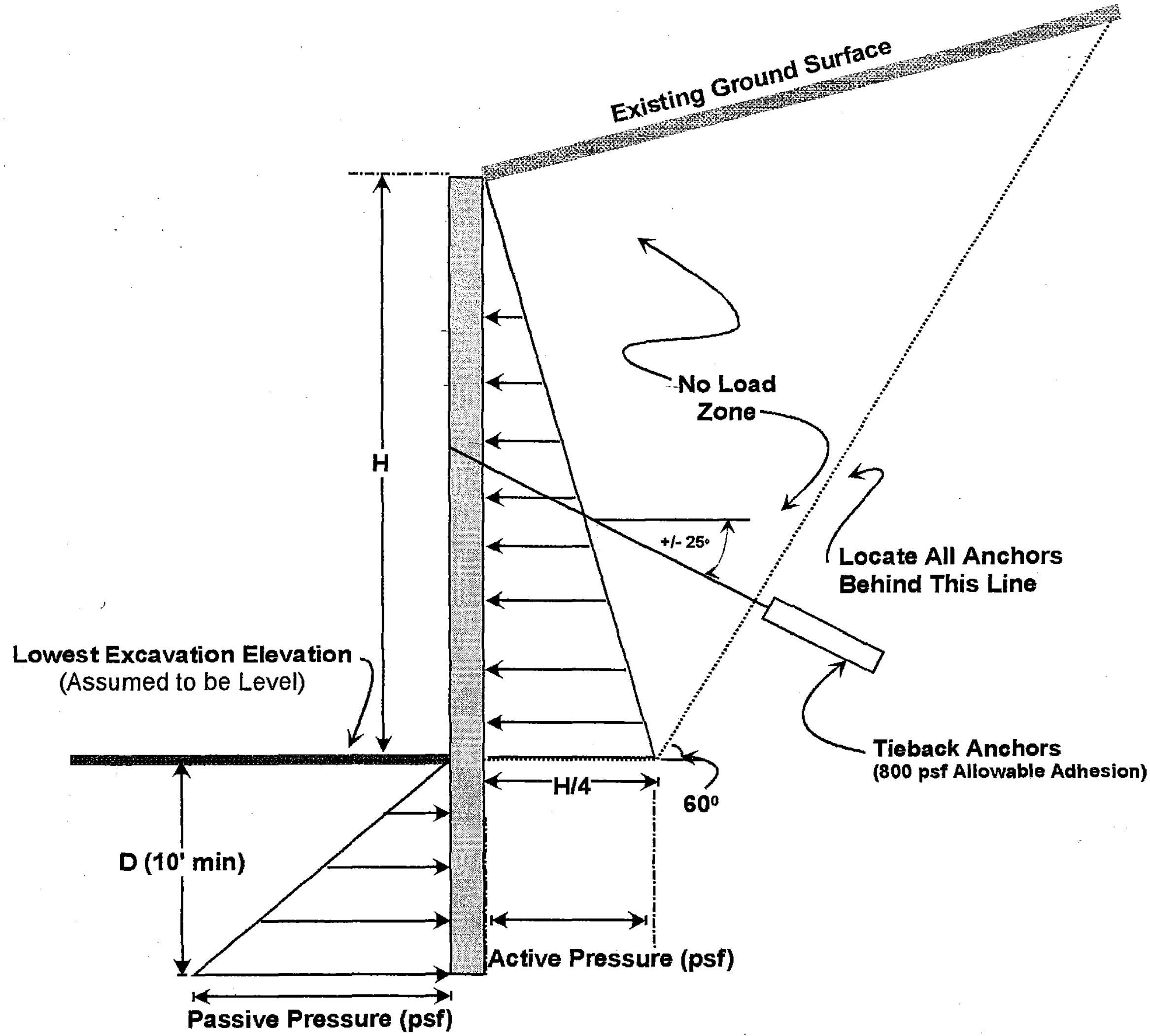
FOOTING DRAIN DETAIL

3100 Airport Way South
Seattle, Washington

Job No:	Date:	Scale:	Plate:
03490	Apr. 2004	Not to Scale	8

RCLLC 0000395



Notes:

(1) The report should be referenced for specifics regarding design and installation.
(2) Active pressures act over the pile spacing.
(3) Passive pressures act over twice the grouted soldier pile diameter or the pile spacing, whichever is smaller.
(4) It is assumed that no hydrostatic pressures act on the back of the shoring walls.
(5) Cut slopes positioned above or behind shoring will exert additional pressures on the shoring wall.

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TIED-BACK SHORING DETAIL
3100 Airport Way South
Seattle, Washington

Job No:	*Date:*		*Plate:*
03490	Apr. 2004		9